APPENDIX B
ACADIANA BAYS REEF RESTORATION PROJECT

GEOTECHNICAL REPORT – LOURIE CONSULTANTS

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LOURIE CONSULTANTS

GEOTECHNICAL ENGINEERING SERVICES
DNR CONTRACT NO. 2503-02-26
ACADIANA BAYS FEASIBILITY STUDY
COASTAL LOUISIANA

REPORT NO. 0102-0011
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This report is a conceptual design document that summarizes our geotechnical activities for this project. In general, these services have consisted of reviewing historical data, conducting various research activities, attending project meetings, interacting with other team members, and performing certain geotechnical engineering analyses. We have provided our services in general accordance with the provisions contained in the contract and agreement executed by Lourie Consultants and Waldemar S. Nelson and Company, Inc. (NELSON) that has an effective date of July 1, 2003. On February 12, 2006, we submitted a draft copy of this report to you for review. This final report supersedes all previously furnished data and completes our involvement with this phase of the project.

Project Overview

The Acadiana Bays area of Louisiana consists of those bays in the central part of coastal Louisiana including from east to west – Four League, Atchafalaya, East Cote Blanche, West Cote Blanche, and Vermilion Bays. The primary goal of this project is to re-establish many of the ecosystem functions of the historical reefs in this area. The Louisiana Department of Natural Resources (LDNR) plans to do this by incorporating, to the extent practicable, natural means of limiting the significant flow of fresh water from the Atchafalaya River, Wax Lake Outlet.

To accomplish the project goal, the LDNR has retained NELSON to conduct a feasibility study to identify and evaluate alternatives. In general, the alternatives being considered to accomplish the project goal are constructing a rubble-mound structure that begins near Point Chevreuil and extends about 13 mi west into East Cote Blanche Bay or constructing a system of staggered oyster reefs in East Cote Blanche Bay.

Purposes and Scope

Our role on this project was to provide geoscience (geology and geotechnical engineering) services to NELSON and its design team. Although we proposed to conduct project- and site-specific geotechnical field and laboratory testing services as part of our project involvement, those services were not authorized. In general, our authorized scope of services consisted of:

- attending project meetings with the LDNR and NELSON team in Baton Rouge, as well as attending project meetings with NELSON personnel and/or various members of the NELSON team
- collecting and evaluating relevant data from various sources, including reports generated for the general project area that were provided to us by the LDNR, information from other members of the NELSON team, and data in our library
- conducting telephone interviews with personnel who were believed to have experience with similar projects

- preparing this report that summarizes our involvement, describes and discusses the results of our analytical efforts, and presents our thoughts about the existing data and the need for additional site- and project-specific geotechnical data

The analytical activities that we have conducted are based on our use and interpretation of data obtained by others for unrelated projects in the general area. We have supplemented that information with a limited amount of research of published literature, telephone interviews, and our experience.

Report Summary

We conducted a geotechnical study specifically for the Acadiana Bays Feasibility Study in the shallow waters of coastal Louisiana. Based on our interpretation of the data collected for this study and our various analyses, we have summarized our findings and conclusions below.

- The field and laboratory data made available to us were collected for other projects onshore or close to the northern shores of the Cote Blanche Bays, Vermilion Bay, or in the Gulf of Mexico, but not near Point Chevreuil or in the study area.
- Soil borings and laboratory data indicate that we should expect very soft to soft, moderately to highly plastic compressible clay deposits in the project area. Near the coastline, these Holocene (Recent) clay deposits usually have thicknesses of about 15 to 25 ft and they tend to deepen toward the Gulf of Mexico. Therefore, in the project area, we expect them to have a thickness of about 40 ft and then be underlain by older, stronger, and generally less compressible Pleistocene deposits.
- In general, the existing geotechnical data indicate the Recent clays are approximately normally consolidated with respect to current overburden pressures, and their undrained shear strengths tend to increase linearly with increasing depth.
- These relatively thick, low-strength, compressible clays make bearing capacity, slope stability, and settlement important design, construction, and maintenance issues for a rubble-mound structure or any other any type of equivalent, constructed feature.
- As an alternative to building on the clays in their current condition, we identified an *in-situ* soil improvement technique that has the potential to improve the strength and reduce the compressibility of the existing soils, which should simplify design, construction, and maintenance of a rubble-mound (or equivalent) structure.
- All of the analyses that we conducted are based on *assumed* soil conditions and engineering properties. A detailed site- and project-specific geotechnical study is needed to refine the concepts discussed in this report and to produce a safe, cost-effective design.

The remainder of this report contains more detailed descriptions of our project involvement, assumptions, and analytical activities. Therefore, the above summary must be used in conjunction with the remainder of this report.

Report Format

The initial sections of this report contain details of the field and laboratory phases of our study. Following these sections, we present a description of the site and subsurface conditions. Next, we present our findings and recommendations regarding design and construction of your proposed project. The illustrations developed for this study follow the text and complete this report.

Limitations

This report is a conceptual design document that we have prepared for the specific project and location described here. We have conducted this study using the standard level of care and diligence normally practiced by recognized engineering firms now performing similar services under similar circumstances. We intend for this report, including all illustrations, to be used in its entirety. The following paragraphs describe other aspects about the use of this report.

Subsurface Conditions. This report describes our assumptions and conclusions about possible subsurface conditions along the project alignment. We have based our assumptions and interpretation of the soil conditions on data obtained from reports prepared by others for unrelated near shore and offshore projects near the central Louisiana coastline. None of the subsurface information that we used in formulating our opinions and conclusions was collected for this specific project or along the proposed structure alignments. In addition to the data from previous studies, we considered the general geologic environment and we relied on our geotechnical engineering experience and professional judgment. It must be recognized that detailed engineering analyses and project designs require project- and site-specific studies to characterize subsurface conditions and to assess the aerial and vertical variations that can occur. Therefore, the actual subsurface conditions along a given project alignment are likely to vary from those discussed in this report.

Use of Data. We have prepared this report for the exclusive use of our Client (Waldemar S. Nelson and Company, Inc.), NELSON team members, and the Louisiana Department of Natural Resources (LDNR). We developed our scope of services and prepared this report based on conversations and the exchange of information between Lourie Consultants and these entities. This information includes details about project goals and concepts, as well as the consequences of using the limited geotechnical data available to us to develop a simplified subsurface soil model for use in our analyses. Because of the importance of these project-related communications, third parties should not rely on the results of this study. Any third parties that do rely on this report do so at their own risk. All parties that use this report will be bound by all of the contract conditions and report limitations that exist between Lourie Consultants and its Client for this project. In addition, third parties are specifically excluded from becoming a third-party beneficiary to Lourie Consultants' contract with its Client. The purpose of this report is to evaluate the conceptual design of the project as it relates to our interpretation of the geotechnical aspects discussed here. This complete report should be available for information only and not as a warranty of subsurface conditions. All parties that use this report must recognize that the report was not prepared for purposes of detailed design, bid development, or construction. These uses all require additional study to obtain the type of information required for those purposes.

Project Meetings and Other Communications

During the course of the project, we attended five meetings at the LDNR offices in Baton Rouge, three meetings with NELSON and/or other team members in New Orleans, and one meeting with NELSON personnel in New Iberia. The purposes of these meetings were to:

- review and refine the project goals
- discuss in general terms some broad concepts for possible structure types (*i.e.*, reefs and jetties)
- obtain existing data and reports collected by others from the project vicinity
- discuss currently identified data needs, including the planned data acquisition program; no new geotechnical-related field or laboratory data acquisition activities have been authorized to date
- exchange information and update the NELSON team and the LDNR about our findings and the results of our analyses
- review and revise the project schedule, including establishing future project and public meeting dates

In addition to the nine project meetings, we also discussed the project and exchanged information during telephone calls and in e-mails.

Geotechnical Information Sources

During a project meeting with LDNR personnel at the outset of this study, LDNR personnel provided us with several project reports. In addition to the LDNR-provided reports, we asked NELSON team members to review their project files to identify other possible sources of information that may have been developed for pipeline route studies, offshore platforms, and other structures. Although, no additional data were located from this effort, we were able to supplement the information with a report(1) from our library that contains a significant amount of high-quality offshore geotechnical information.

Project Reports. Listed below are the reports that we have reviewed to obtain geotechnical information from the general project area.

- *A Jetty from Point Chevreuil: An Evaluation of a Proposal to Reduce Sedimentation in the Cote Blanche Bays and Vermilion Bay,* Coastal Environments, Inc., June 1977.
- *Geotechnical Investigation, U.S. Department of Agriculture, Cote Blanche Hydrogeologic Restoration. St. Mary Parish, Contract No. NRCS-6-LA-203,* Eustis Engineering, Company, Inc., Job No. 14120, January 20, 1997, (LDNR Project No. TV-04).

(1) McClelland Engineers, Inc., (1979), *Strength Characteristics of Near Seafloor Continental Shelf Deposits of North Central Gulf of Mexico,* Report No. 0178-043, Houston, TX.

- *Report of Geotechnical Services, Coastal Restoration Project, Little Vermilion Bay,* Soils and Foundation Engineers, Inc., SFE Job No. 97-213, March 9, 1998, and *Supplemental Analyses, Little Vermilion Bay, Coastal Restoration Project,* Soils and Foundation Engineers, Inc., SFE Job No. 97-213, March 30, 1998, (LDNR Project No. TV-12).
- *Four Mile Canal Terracing and Sediment Trapping, Engineering and Design Services, Vermilion Parish, Louisiana, DNR Contract No. 2511-02-09, OCR Contract No. 435-200394,* Soils and Foundation Engineers, Inc., SFE Job No. 01-154, June 4, 2002, (LDNR Project No. TV-18).
- *Geotechnical Investigation, Sediment Trapping at the JAWS Project (TV-15), Teche/Vermilion Basin, West Cote Blanche Bay, St. Mary Parish, Louisiana,* Soils and Foundation Engineers, Inc., SFE Job No. 99-164, September 14, 1999, and *Results of Stability Analyses, DNR Sediment Trapping at the JAWS (TV-15),* Soils and Foundation Engineers, Inc., SFE Job No. 02-158, September 20, 2002, (LDNR Project No. TV-15).
- *Mississippi River and Tributaries, Atchafalaya Basin, Louisiana, Lower Atchafalaya Basin Reevaluation Study,* US Army Corps of Engineers, Mississippi River Commission, New Orleans District, October 2002.
- *Freshwater Bayou Bank Stabilization, Vermilion Parish, Louisiana,* US Army Corps of Engineers, New Orleans District, 2001 and 2003, (LDNR Project No. TV-11).

Other Searches. After completing the data summary activities described later in this report, we conducted other searches to supplement the sources of information listed above and to obtain information that could be applicable to this project. We used the Internet to search for reports or studies applicable to the design and construction of coastal structures in similar settings. While the river-dominated deltaic islands of the Mississippi River Delta are widely studied, their formation and evolution are unique. Furthermore, most other locations where coastal restoration projects have occurred appear to have granular foundation soils or if very soft clays are present, they are relatively thin, which contrasts with the soil conditions in much of coastal Louisiana.

We contacted the US Army Corps of Engineers (USACE) in Vicksburg, Mississippi, and conducted telephone interviews with Mr. Dick Peterson. During the interview process, we discussed our assumptions and planned analytical approach, and we obtained his insights into foundation design and construction in a marine environment. He also provided us with references, many of which we had already reviewed, including a publication(2) he co-authored. Mr. Peterson agreed with our approach and general conclusions that where thick deposits of soft soil are present, structure loads will have to be limited unless foundation soil replacement (including displacement) and/or soil improvement occurs. In some cases and for certain types of structures, piles or other deep foundations can be used, but these solutions would not generally be applicable to a long, linear coastal restoration/protection structure.

After completing our initial analyses for the conceptual design of a rubble-mound structure on the unimproved seafloor soils, NELSON prepared preliminary cost estimates, as well as pre-

(2) Lee, Jr., L.T. and Peterson, R.W., (2001), *Underwater Geotechnical Foundations*, ERDC/GSL TR-01-24, US Army Research and Development Center, Vicksburg, MS.

sented and discussed them with LDNR personnel. Following the presentation and after additional discussions with NELSON personnel, we conducted further research into *in-situ* soil improvement methods that could be applicable to relatively thick deposits of very soft and soft clays in an off-shore or a shallow water marine environment. This additional research indicated that one or more of the various deep soil mixing methods (DSMMs) could be suitable for treating and improving the engineering properties of the foundation soils *in situ*. While there are several patented and competing methods, DSMMs involve forming a series of treated soil columns by using multiple mixing shafts guided by a crane-supported set of leads. As the mixing shafts are advanced into the soil, a 'binder' (an admixture such cement, lime, flyash, etc.) is usually pumped through the hollow stem of the shaft and injected into the soil at the tip. The auger flights and mixing blades on the shafts blend the soil with the binder in pugmill fashion. The mixing shafts are positioned to overlap one another and form a continuously mixed overlapping column. When the design depth is reached, the augers are withdrawn and the mixing process is repeated on the way up to the surface. The resulting and improved soil material generally has a higher strength and lower compressibility than the native soil, although the total unit weight may be less. The exact properties obtained reflect the characteristics of the native soil, the construction variables (primarily the mixing method), the operational parameters, and the binder characteristics.

Geotechnical Data Summary

While some studies have been conducted over the years in the bays included in the project vicinity, none of the reviewed information contains geotechnical data specifically from the project area. The geotechnical studies listed above were performed onshore or close to the northern shores of the Cote Blanche Bays, Vermilion Bay, or in the Gulf of Mexico, but not near Point Chevreuil or in the study area. Also, the above-referenced Coastal Environments (1977) report is not a geotechnical study, but it does contain some information concerning bottom sediments to about 2- to 3-ft penetration near Point Chevreuil and in the study area.

Although none of the reviewed studies contained geotechnical data that can be used for detailed design or construction purposes, they do contain some limited information about general soil conditions that can be expected to occur in the project area. For design of any structure, it is important to determine the approximate thickness, composition, and critical engineering parameters of the various soil strata present in the specific project area. Important geotechnical engineering parameters of the *in-situ* soils include strength properties (shear strength and friction angle) and soil compressibility characteristics. Therefore, in performing our review, we were primarily interested in characterizing:

- types and distribution of soil present at and below the seafloor
- strength properties of the soils
- compressibility properties of the soils

Using data contained in the above reports, we established a database and populated it with data from LDNR Project Nos. TV-4, TV-11, TV-15, and TV-18:

- Boring summary: 70 borings, surface elevations range from about El +2.5 to about El -21 ft, and penetrations range from 7 to 110 feet.

- Data summary: 281 water contents, 231 unit weights, 296 strength tests, 61 liquid and plastic limit tests, and 10 grain-size tests.

Plates 1 through 5 contains plots of undrained shear strength, unit wet weight, water content, plasticity index, and liquidity index.

Design Considerations, Methods of Analysis, and Recommendations

In this section of the report, we discuss our interpretation of the data. In addition, we also state key assumptions, discuss our methods of analysis, and present the results of our analyses.

General Foundation Design Criteria. Foundation design for any structure must satisfy two basic, independent criteria. First, the pressure transmitted to the foundation soils should not exceed the allowable bearing pressure. This allowable bearing pressure includes an adequate factor of safety (FS) applied to the soil shear strength. Second, settlement of the underlying soils due to consolidation of the foundation soils during the operating life of the structure must be within tolerable limits. In addition to these technical issues, other factors such as construction schedules, weather conditions, project economics, and the owner's performance criteria, can influence final foundation design.

The undrained shear strength of the clays will control bearing capacity (the externally applied load that the soil can support without failure) during and immediately after construction. Compressibility properties of these clays will control the rate and magnitude of settlement that will occur due to the applied loads. For siliceous granular soils, particle size, gradation, and density control bearing capacity and settlement properties. In general, granular soils tend to be stronger (have higher bearing capacities), less compressible (settle less), and more permeable (settle quicker) than cohesive soils.

Assumed and Generalized Stratigraphic Model. Our interpretation of the data contained in the above reports suggests that the predominant soils are likely to be very soft to soft, moderately to highly plastic silty clays and clays (cohesive soils). Some of these soils contain varying amounts of organic matter, wood, shells, and shell fragments. The Holocene (Recent) silty clays and clays have high water contents, are highly compressible, and probably extend to significant depths below the seafloor where they are underlain by older Pleistocene soils. Compared to the overlying Recent sediments, the Pleistocene sediments often tend to be stronger, have lower water contents, and are appreciably less organic. The Pleistocene sediments also are likely to be less compressible. Occasionally, partings, seams, and layers of granular soil (silts, sands, and silt-sand mixtures) may be present within predominantly cohesive soil deposits. In some localized areas, granular soil strata that are thicker and have a greater aerial extent may also be present.

The available data indicate the very soft to soft silty clays and clays in the area are likely to be approximately normally consolidated with respect to current overburden pressure, and have undrained shear strengths of about 50 psf or less at the seafloor that increase slowly with depth to the top of the underlying Pleistocene surface. The near shore studies indicate that these very soft and soft Recent soils extend to at least 15- to 25-ft penetration and often extend to about 40- to 50-ft penetration below the mudline. At the base of these Holocene deposits, the undrained shear strengths appear to be as low as about 100 psf to perhaps as high as 300 to 400 psf. Data from the offshore area (McClelland's 1979 report) suggest that the very soft and soft soils are likely to be present to about 60-ft penetration below the seafloor.

After a project meeting with NELSON personnel, we reviewed transects obtained by Fugro-Chance and plotted by NELSON. In the area that is being considered for a rubble-mound structure, Transects 15 through 17 and 19 show the following seafloor elevations, which are in feet and are referenced to North American Vertical Datum, 1988 (NAVD 88):

- Maximum Range: El -2.59 to -6.14
- Minimum Range: El –10.70 to -14.73
- Average Range: El -7.81 to -10.60

Geotechnical Analysis: Unimproved Soils. Due to the preliminary nature of this study, the limited amount of available geotechnical data, and the large number of assumptions that we had to make, we elected to begin our analysis using bearing capacity methods rather than more complex slope stability methods. Bearing capacity methods are useful for estimating the potential for weak, saturated, clay foundations to support embankments such as a rubble-mound structure. The method compares the ultimate bearing capacity of the foundation beneath the structure to the total vertical stress imposed by the structure. The vertical stress is calculated by multiplying the full height of the rubble-mound structure by the total unit weight of the fill material. This simplified approach has limitations and is not intended to be a substitute for more rigorous slope stability analyses. However, we believe this approach provides a relatively quick and useful method for evaluating the short-term, undrained stability of embankments resting on soft, saturated clay foundations, and it can serve as an approximate check of more rigorous and thorough analyses.

Although there is variability in the data, most of the data plots contained on Plates 1 through 5 show the soils differ above and below about 40-ft penetration, with the soils below about 40 ft being stronger than the shallower soils. The plots also suggest an increase in undrained shear strength with depth. Therefore, to conduct our analyses, we used a bearing capacity procedure[3] that accounts for a linearly increasing strength profile and can account for a range of initial seafloor shear strengths. The equation we used computing the ultimate bearing capacity is as follows:

$$Q_{ult} = F_R(5.14S_{uo}+\rho B/4)(1+0.2D_f/B)(1+0.2B/L) \text{ for } D_f/B \le 2.5 \text{ and } B \ge L$$

where:

Q_{ult} = Ultimate bearing capacity, psf

F_R = Correction factor, a function of ρ, B, and S_{uo}

S_{uo} = Undrained shear strength at the base of the footing, psf

ρ = Rate of increase in undrained shear strength with depth, psf/ft

B = Footing width or diameter, ft

L = Footing length or diameter, ft

D_f = Footing depth, ft

(3) Davis, E.H. and Booker, J.R., (1973), *The Effect of Increasing Strength with Depth on the Bearing Capacity of Clay*, Geotechnique 23 No. 4, pg 551-563, Canada.

Initially, we conducted analyses using a range of undrained shear strengths at the seafloor (mudline) and then allowing the shear strength values to increase linearly with depth to 400 psf at 40-ft penetration. However, for conceptual design purposes, we modeled undrained shear strengths in the project area by assuming a value of 0 psf at the mudline and allowing it to increase slowly and linearly with depth to 400 psf at 40-ft penetration. This implies an increase in undrained shear strength of 10 psf/ft of penetration, which is realistic for normally consolidated, moderately to highly plastic Gulf of Mexico clays. Overall, we believe that the shear strength model and the analytical method that we used for this stage of the project are appropriately conservative given the absence of actual data.

At meetings with NELSON and then with the LDNR, we presented the results of our analyses as a series of plots. Plates 6 through 9 show plots of foundation base width (B) vs effective soil unit weight (γ') for a variety of embankment heights (H), and a minimum computed factor of safety, $FS_{(min)}$, equal to 1.10 for the short-term (end-of-construction) condition. During these meetings, discussions occurred about the selection of an appropriate safety factor, which for simplicity we defined as follows:

$$FS = \Sigma F_r / \Sigma F_d$$

where: ΣF_r = Sum of all resisting forces

ΣF_d = Sum of all driving forces

In the above equation, when FS = 1.0, the forces are in balance; when FS is more than 1.0, the resisting forces are greater (stable conditions); and when FS is less than 1.0, the driving forces are greater (unstable conditions). In general, structures designed and built with low safety factors have a greater likelihood of failure than those with greater safety factors. We noted that the choice of an appropriate safety factor must consider many issues, including the uncertainty in the data and the consequences of failure. Finally, we commented that as the foundation soils consolidate (settle) under the applied loads, the soils would become stronger and stiffer; therefore, long-term factors of safety will increase with time and be greater than the short-term condition.

As a check on the bearing capacity approach discussed above, we also made a few slope stability analysis runs using the computer program PCSTABL and a simplified embankment and subsurface soil model. The PCSTABL computer program is a PC-based, two-dimensional, limit-equilibrium slope stability analysis program developed at Purdue University for the Federal Highway Administration. The program uses random techniques for generating potential failure surfaces for a variety of slope stability methods and several types of failure surfaces. For this limited assessment, we used the Bishop Simplified Method of analysis to evaluate thousands of potential circular failure surfaces and had the program identify the 10 most critical surfaces (the surfaces with the lowest computed safety factors). The most critical surface is the one with the lowest computed FS, $FS_{(min)}$. In general, the PCSTABL results were consistent with the results from the bearing capacity analyses.

Geotechnical Analysis: DSMM-Improved Soils. As discussed above, DSMMs have the ability to improve the engineering properties of soils, including those of soft, compressible marine clays that we expect to be present in the project area. Our limited review of published information about DSMMs indicated that we could expect significant increases in soil strength to occur following a DSMM program. Therefore, during a meeting with NELSON personnel, we discussed the design and construction of a rubble-mound structure on a DSMM-improved soil foundation. To evaluate this concept, NELSON asked us to consider a rubble-mound structure constructed with material having an effective unit weight of 86 pcf (submerged condition) and the following geometry:

- side slopes: 1-vertical on 3-horizontal (1-V:3-H)
- maximum height above the seafloor: 9 ft
- crest width: 8 ft
- maximum base width: 62 ft

Using the above criteria, we used a bearing capacity approach to back-calculate the required soil shear strength of about 150 psf is necessary to produce a FS of at least 1.0 for an infinitely long footing constructed on the seafloor. The equation we used for this purpose was:

$$Q_{ult} = 5S_u(1+0.2D_f/B)(1+0.2B/L) \text{ for } D_f/B \leq 2.5 \text{ and } B \geq L$$

where: Q_{ult} = Ultimate bearing capacity, psf

S_u = Undrained shear strength below the base of the footing, psf

D_f = Footing depth, ft

B = Footing width or diameter, ft

L = Footing length or diameter, ft

Of course, to obtain a greater value for the FS, the minimum required soil strength would have to be greater. In addition, to reduce the potential for edge-type failures to occur below the toe of the rubble-mound, the width of the improved soil mass would need to be somewhat greater than the base width of the rubble-mound. Finally, based on bearing capacity considerations and experience, the depth to which the minimum required soil strength needs to extend is about 50 percent of the base width of the rubble-mound structure, or about 30 ft below the seafloor for the geometry described here.

Again, as a check on the bearing capacity approach, we analyzed the above rubble-mound structure using PCSTABL. The results were generally consistent, although to obtain the same FS value when DSMM-improved soils are present, a slightly lower average shear strength appears to be required than indicated by the bearing capacity approach. In addition, the slope stability analyses suggest that the soils to about 20- to 25-ft depth influence the location of the critical failure surface below the seafloor. Finally, the critical circles typically extend about 9 to 14 ft beyond the toe of the slope.

Foundation Settlement. A detailed settlement analysis was beyond the scope of this study. In addition, since there are no site-specific soils data, the necessary engineering properties are not available to make detailed calculations about the amount and rate of settlement.

Based on the assumed soil conditions and commonly used correlations between routine soil properties and soil compressibility, we expect the unimproved foundation soils to be quite compressible and subject to significant movements. These movements include initial movements that occur during construction as loads are applied and consolidation-related movements that will occur over the life of the structure. As a very rough approximation, we estimate that for a large loaded area such as one imposed by a rubble-mound structure, about 0.5 ft of settlement can occur for every 100 psf of applied load up to an applied load of about 1000 psf. For example, with an applied load of about 600 psf, we would expect consolidation settlement on the order of about 3 feet. Furthermore, given the expected thickness and low permeability of the *in-situ* soils, we would expect these settlements to occur for an extended period of time. As a result, we anticipate that it would be necessary to place additional fill periodically on the rubble-mound structure throughout its life in attempt to maintain its original height.

For DSMM-improved foundation soils, a soil improvement scheme that increases the strength of the foundation soil, it should also greatly reduce the long-term soil compressibility. Therefore, we would expect that long-term foundation-related maintenance for a structure constructed on DSMM-improved soil would be much less than the maintenance required for a structure over the unimproved foundation soils.

Additional Considerations

The discussions presented above are, by necessity, somewhat general and the analytical approaches and results are approximate. However, we believe that they provide a framework for conceptual design. Presented below are our thoughts about some of the many other factors to consider.

Constructability. In general, construction operations in a marine environment involve equipment and methods that differ from those used for land-based construction. In addition, the construction of rubble-mound structure on the unimproved foundation soils is likely to present a number of challenges because of the expected low strength and compressibility of the foundation soils. During construction on very soft to soft clays, there is the potential for bearing capacity failures and/or large deformations to occur. As such, design and construction practices must consider these possibilities. For example, to reduce the potential for bearing capacity failure, material placement has to occur slowly enough and in stages to allow excess pore pressures generated during material placement to dissipate. Conceptually, we think this type of construction would involve the following:

- placing an anchored or weighted high-strength geotextile (or more likely a high-modulus geogrid) on the seafloor
- constructing a granular (sand) pad on the geotextile/geogrid to displace some of the almost-liquid seafloor soils that we assume are present and to provide a working table on which to build the planned structure
- filling prefabricated geotextile tubes (geotubes) with dredged and hydraulically placed granular material and placing the geotubes parallel to the longitudinal axis of the structure to form stabilizing berms on the gulf and land sides of the planned structure; these geotubes also would act serve as retention dikes for later hydraulic placement of additional dredged material
- capping the geotubes and dredged materials with rip-rap and/or some other type of suitably protective armor

In our opinion, the DSMM-improved soil option has the ability to address many of the concerns and solve many of potential problems associated with attempting to construct a structure on unimproved very soft and soft clay soils. A construction program involving one of the various DSMMs is likely to consist of the following:

- treating the soil to the required depth on some type of predetermined pattern or spacing of columns using a project-specific binder formula
- conducting testing during the DSMM process and after its completion to confirm that the desired soil properties have been obtained
- constructing the planned structure using techniques that are applicable to building like structures on similarly stable seafloor soils

We believe that with proper design and construction, the DSMM approach would:

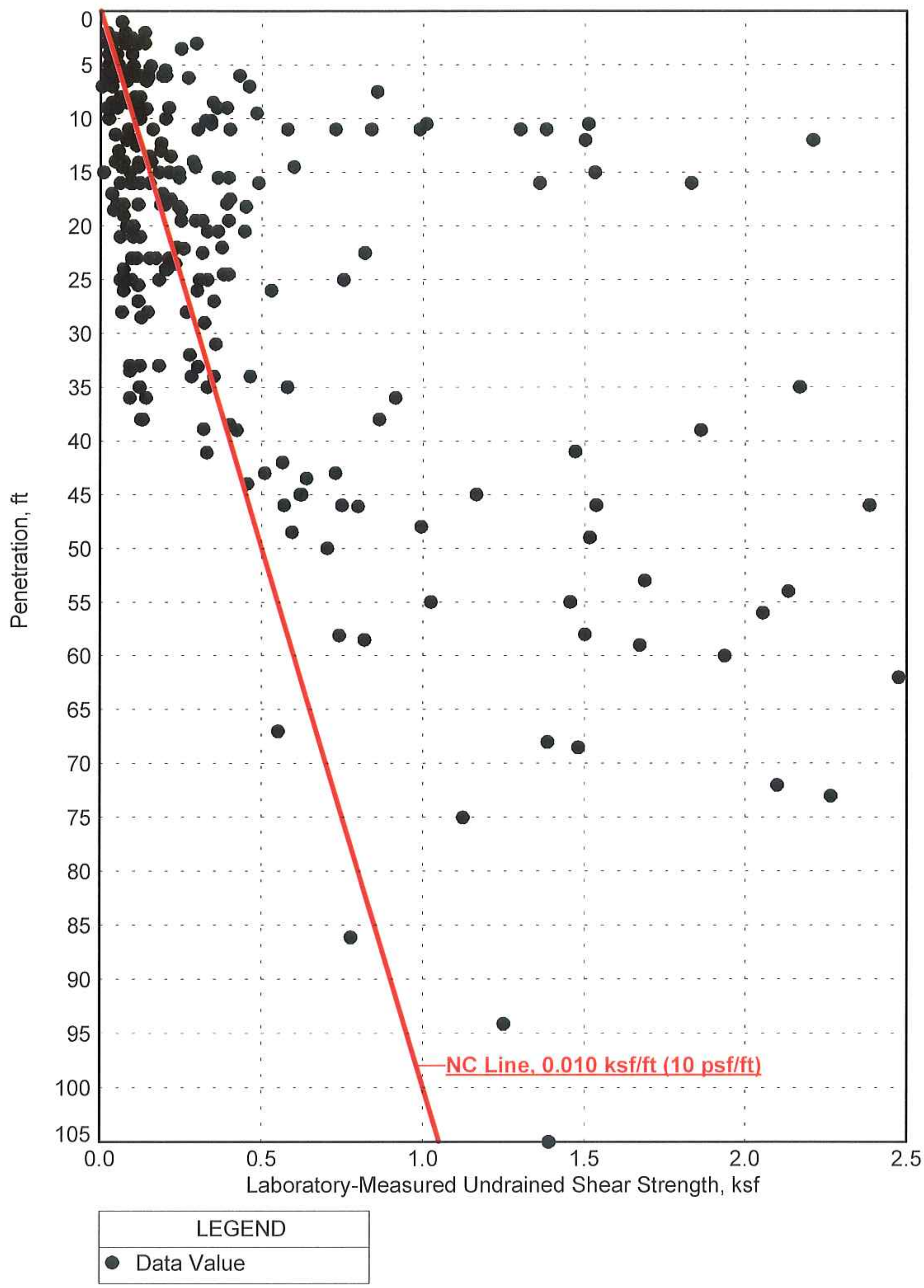
- reduce the potential for failures and/or large deformations to occur during material placement
- allow for more flexibility in selecting materials used to construct the structure
- simplify material placement and allow for more rapid fill placement
- produce a structure that will require less settlement-related maintenance over the life of the structure

Future Geotechnical Data Acquisition and Engineering Needs. To advance the design process using either unimproved foundation soils or some type of improved foundation material additional geotechnical activities need to be undertaken. In part, this includes conducting, site-specific and high-quality field and laboratory studies. These studies are needed to characterize accurately the foundation conditions along the project alignment. Since site exploration identifies subsurface conditions *only* at those points where subsurface tests are conducted or samples are taken, good and customary engineering design practice requires obtaining project- and location-specific geotechnical information. In general, the more comprehensive a study is, the greater the reduction in uncertainty during the design, construction, and operating life of a structure.

A comprehensive and integrated geoscience exploration program could include geophysical studies combined with traditional and modern geotechnical methods. With input from other team members, we could design a geotechnical investigation that would include collecting seafloor and shallow sediments for use by other members of the design team. The nature of the exploration program and the spacing of the exploration points depend on a number of factors. These include the type and size of the proposed structure, the anticipated nature of the soils, and the implications of the *in-situ* soil conditions on the project's feasibility and design concepts.

Following the completion of the field and laboratory programs, data interpretation, parameter selection, and detailed engineering analyses must occur. We would expect that several subsurface profiles would be appropriate for the type of long, linear structure under consideration. Engineering analyses would need to include bearing capacity, internal and external slope stability analysis, and settlement analysis. In our opinion, high-quality field and laboratory programs followed by comprehensive engineering analyses are required especially if cost-effective designs are desired.

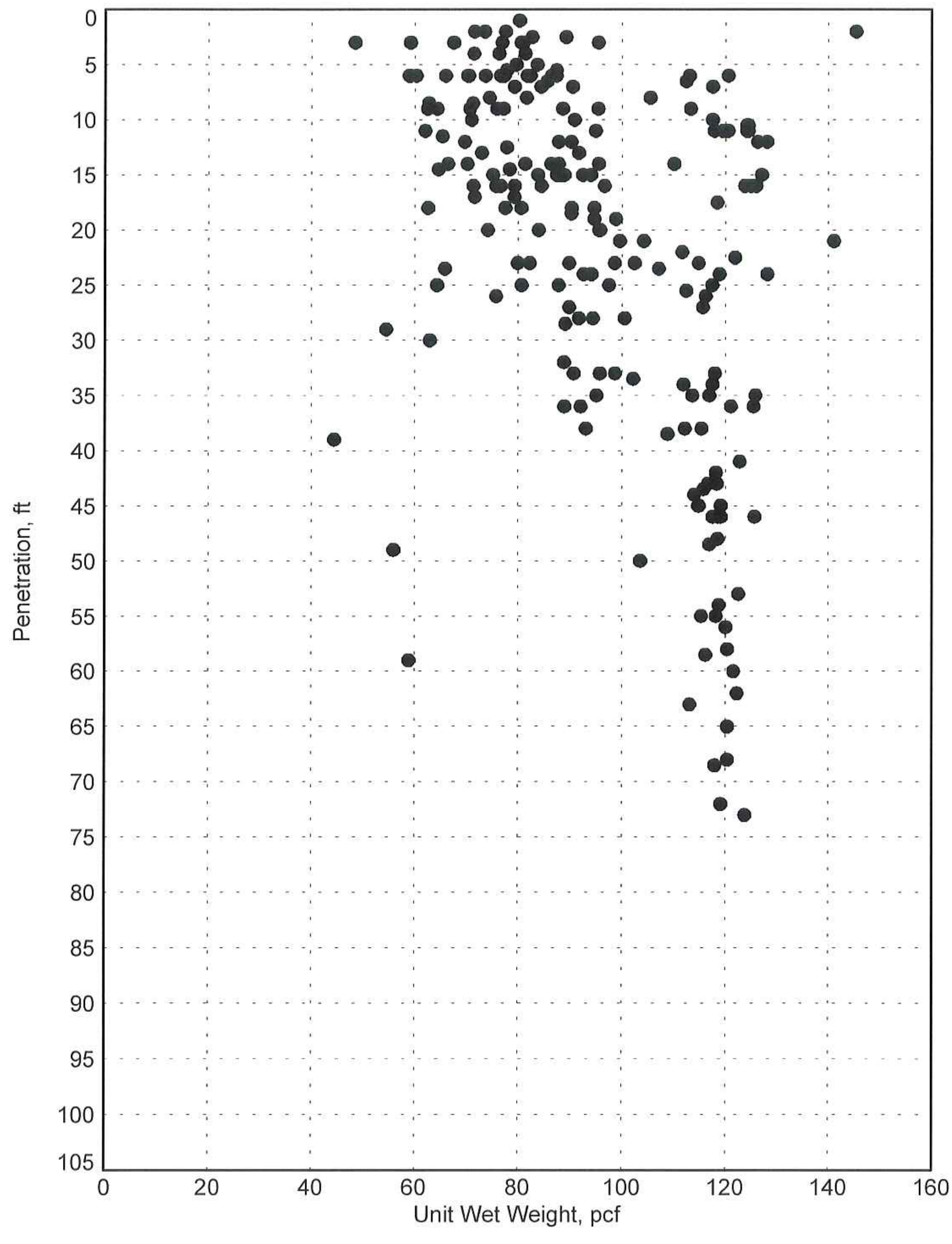
ILLUSTRATIONS



LCUSSLABPLT Rev: 07/19/04 20011.GPJ LOURIE.GDT 02/12/06

UNDRAINED SHEAR STRENGTH PROFILE -- TV-04, -11, -15, AND -18

DNR Contract No. 2503-02-26 -- Acadiana Bays Feasibility Study
Coastal Louisiana

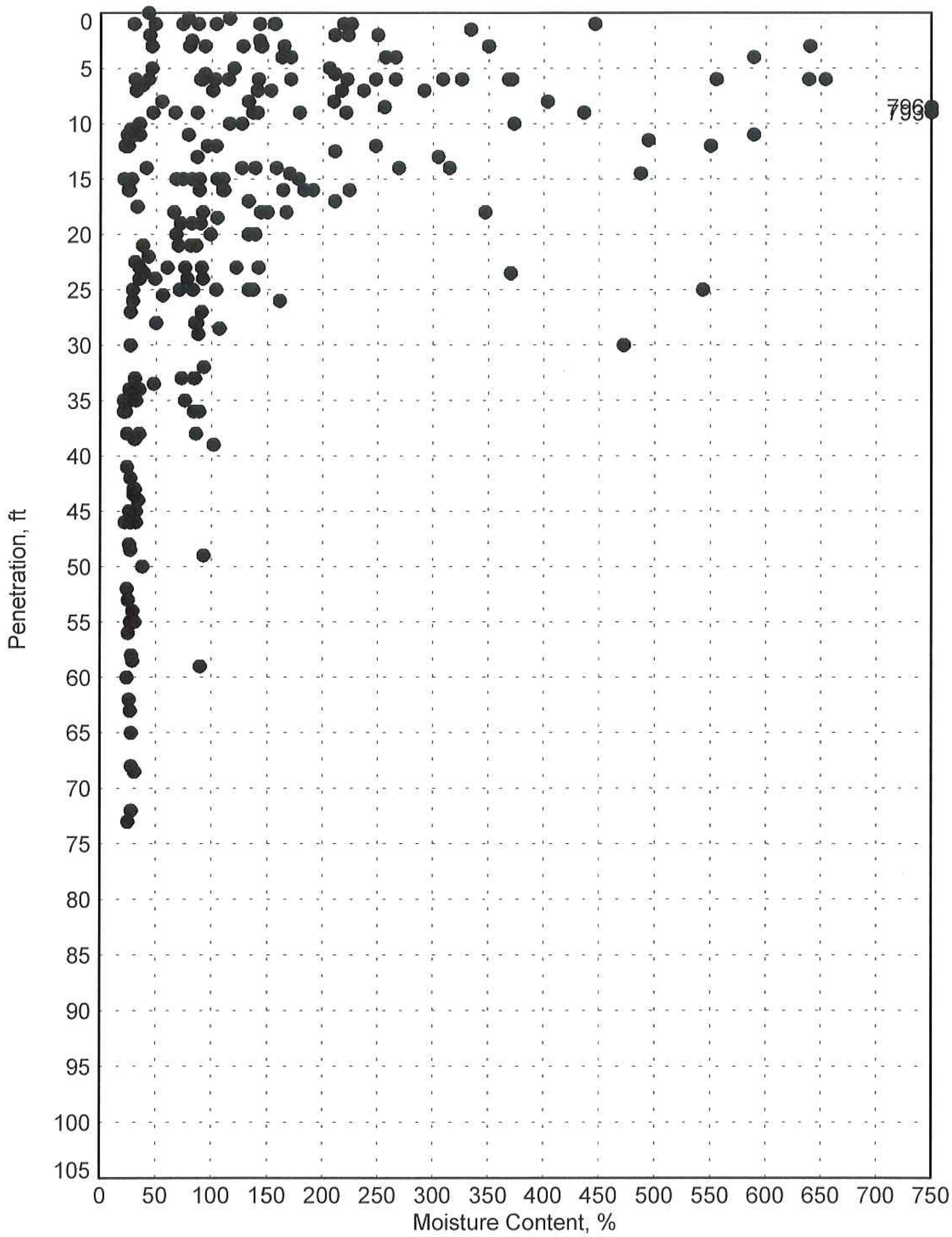


LCUWWPLT Rev: 07/19/04 20011.GPJ LOURIE.GDT 02/12/06

UNIT WET WEIGHT PROFILE -- TV-04, -11, -15, AND -18

DNR Contract No. 2503-02-26 -- Acadiana Bays Feasibility Study

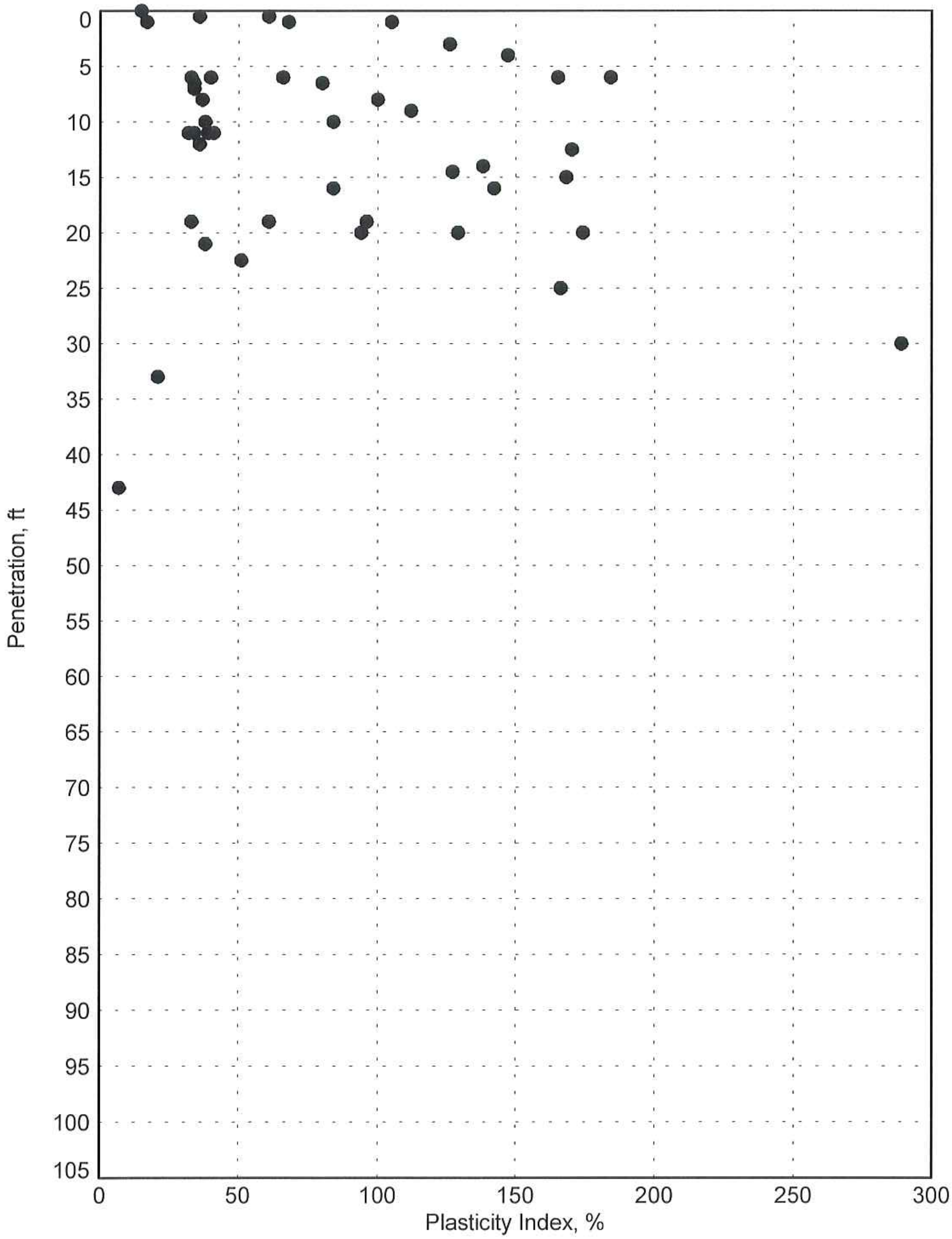
Coastal Louisiana



MOISTURE CONTENT PROFILE -- TV-04, -11, -15, AND -18

DNR Contract No. 2503-02-26 -- Acadiana Bays Feasibility Study

Coastal Louisiana

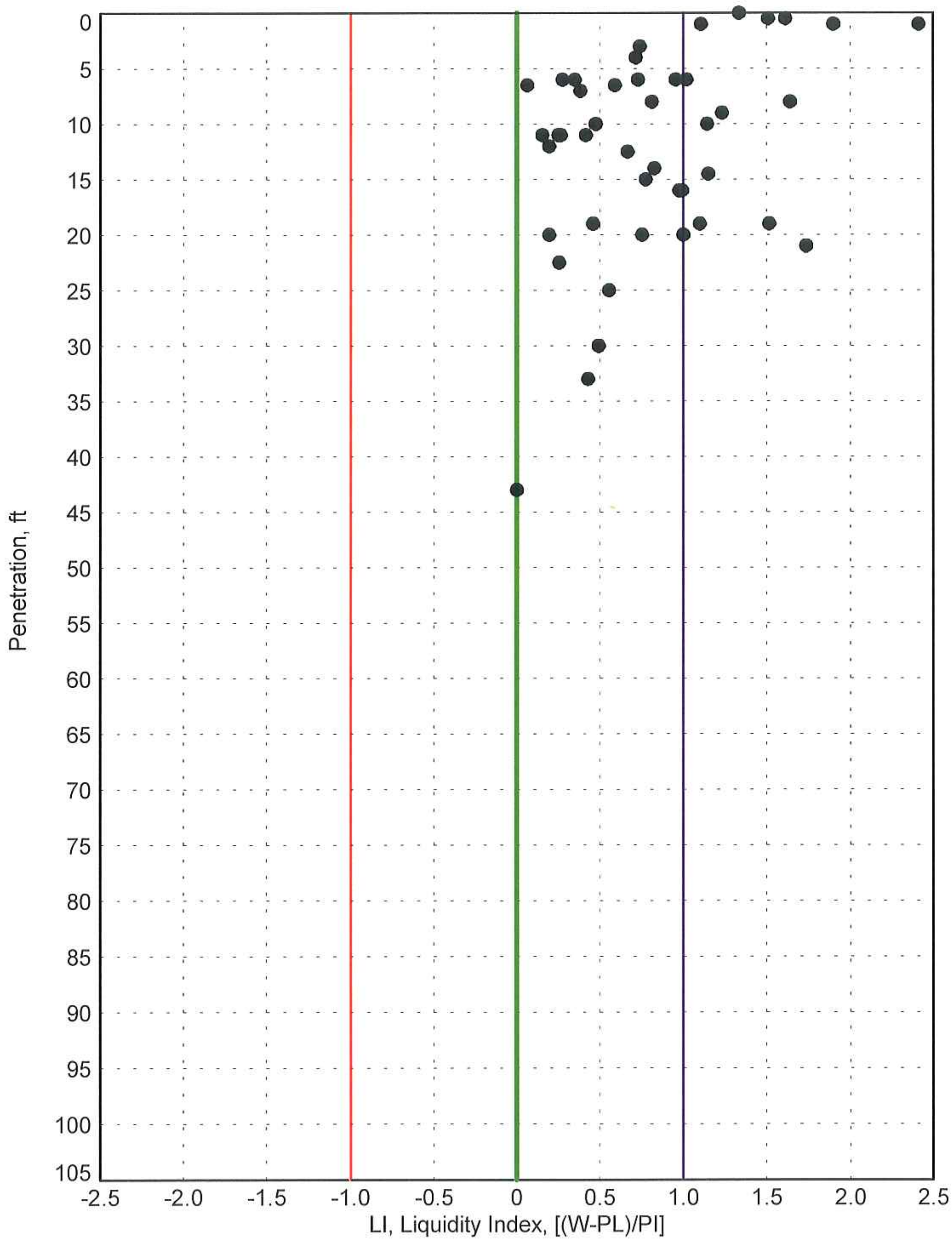


LCPIPLT Rev: 03/19/01 20011.GPJ LOURIE.GDT 02/12/06

PLASTICITY INDEX PROFILE -- TV-04, -11, -15, AND -18

DNR Contract No. 2503-02-26 -- Acadiana Bays Feasibility Study

Coastal Louisiana



LCLIPLT Rev: 01/19/01 20011.GPJ LOURIE.GDT 02/12/06

LIQUIDITY INDEX PROFILE -- TV-04, -11, -15, AND -18

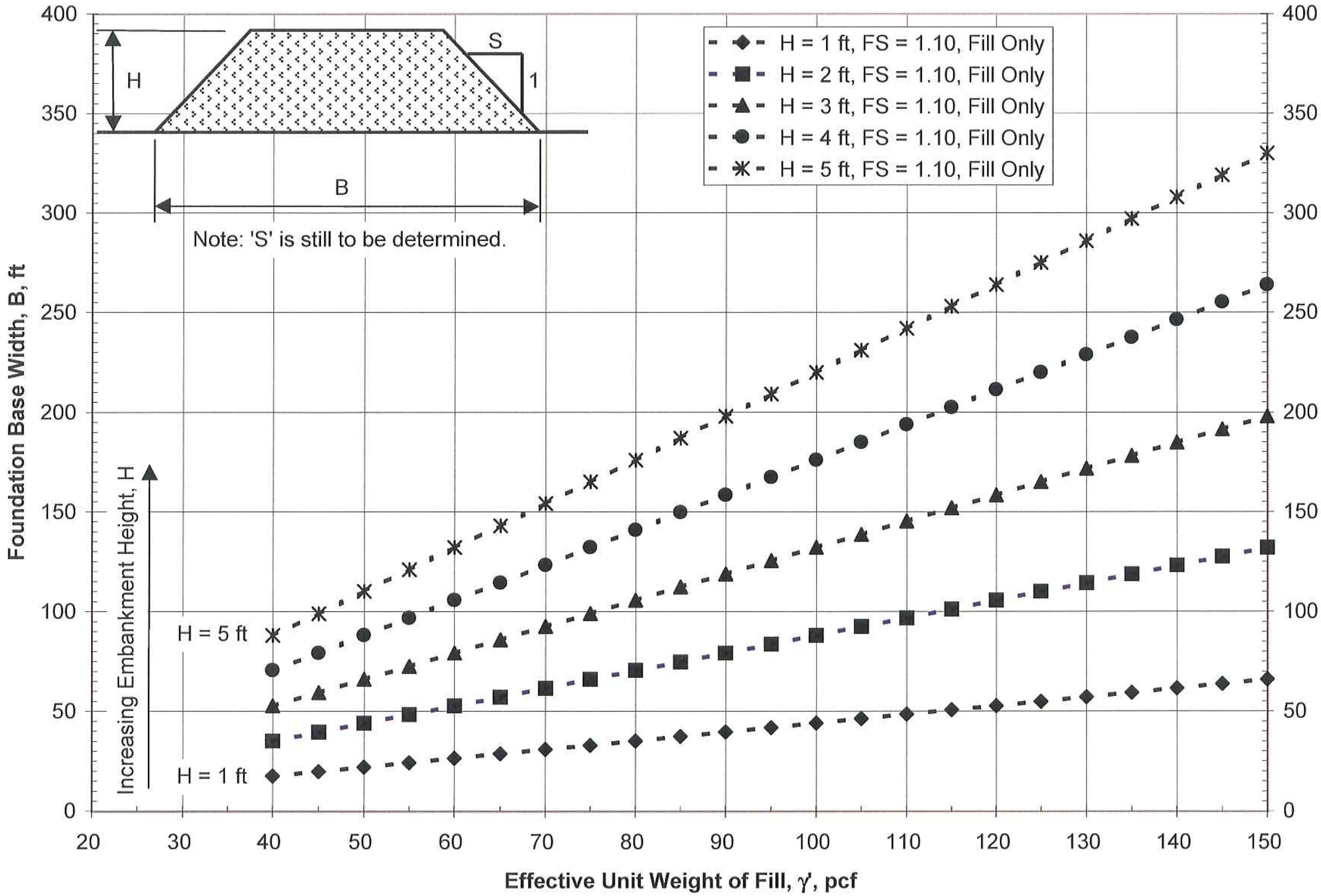
DNR Contract No. 2503-02-26 -- Acadiana Bays Feasibility Study

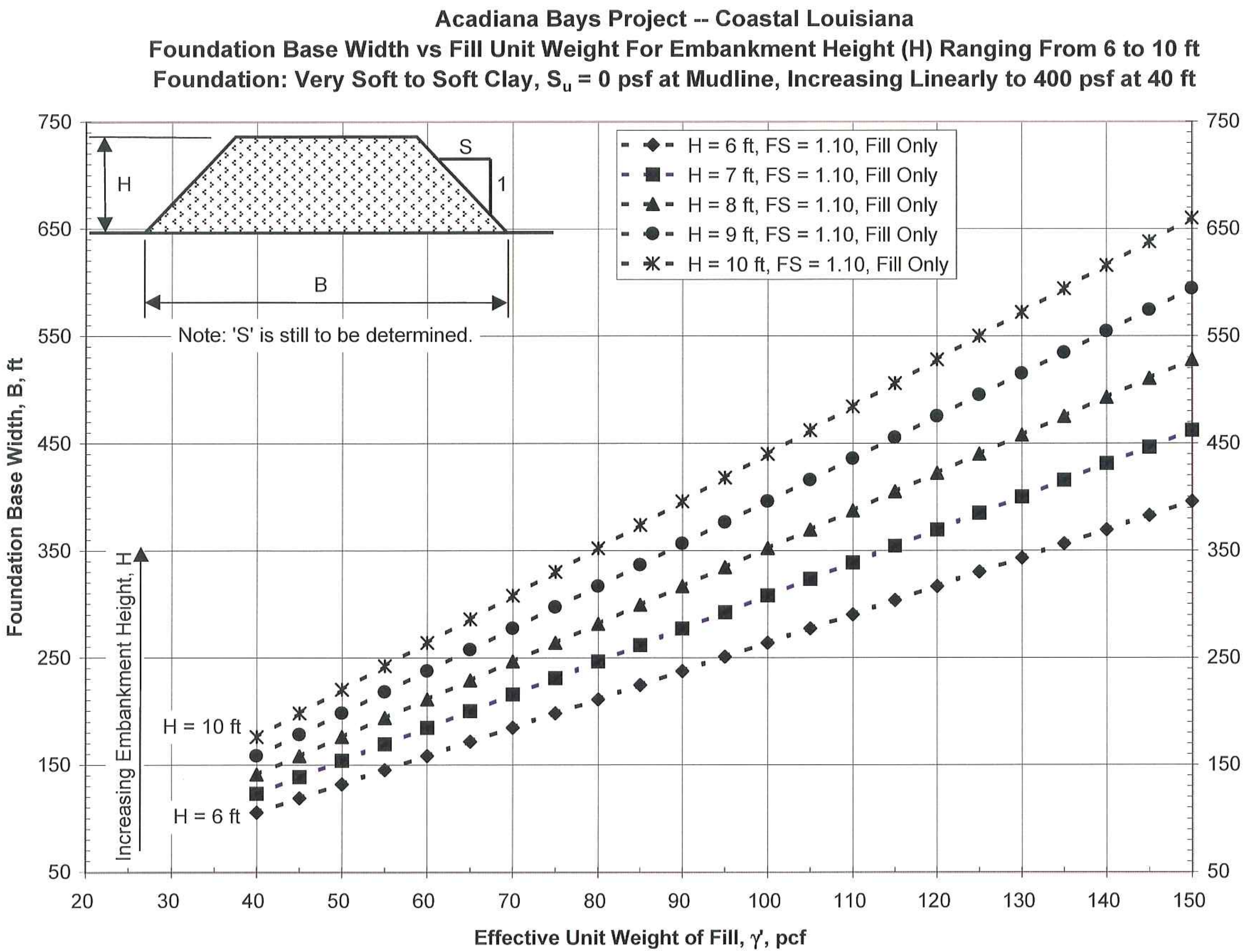
Coastal Louisiana

Acadiana Bays Project -- Coastal Louisiana

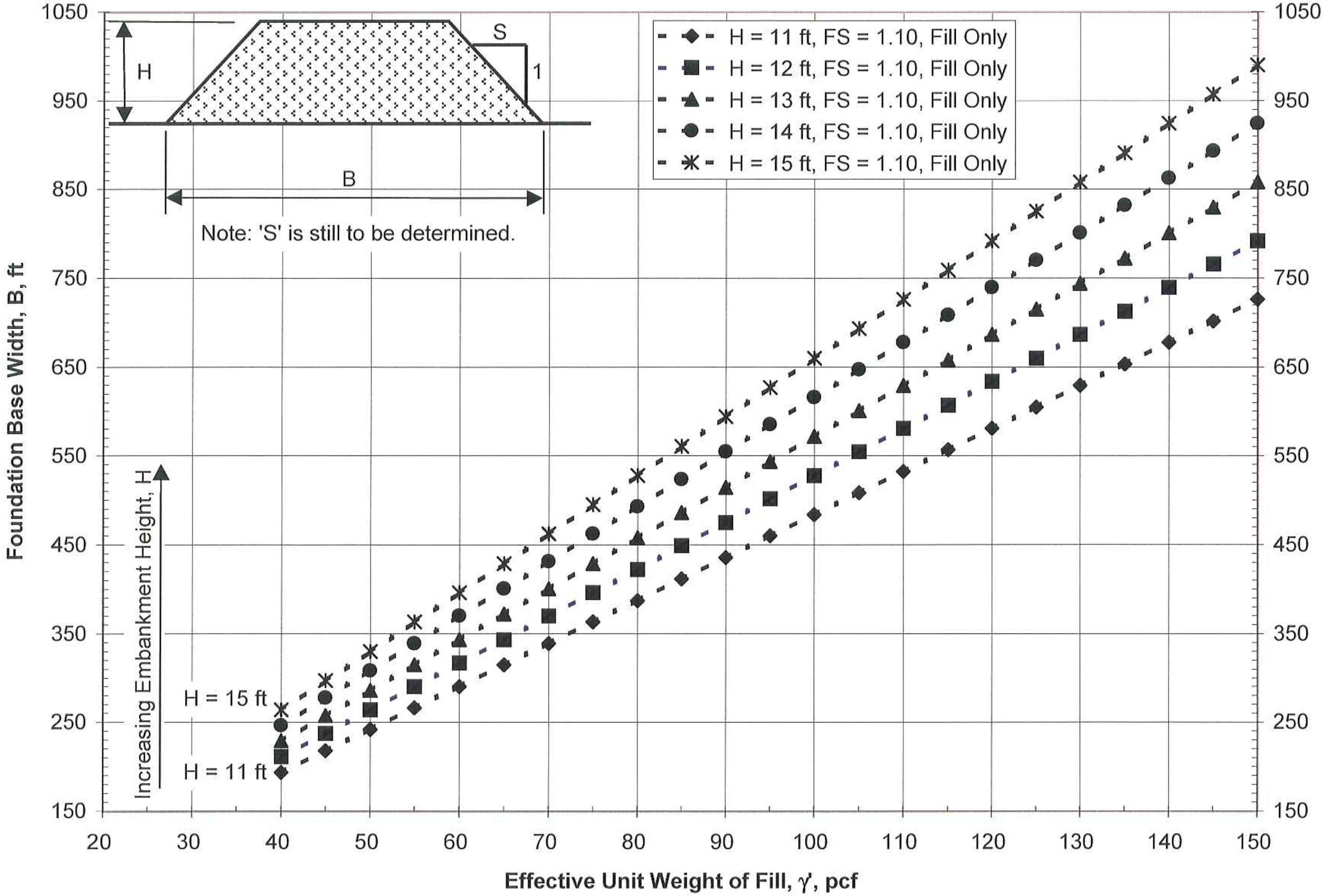
Foundation Base Width vs Fill Unit Weight For Embankment Height (H) Ranging From 1 to 5 ft

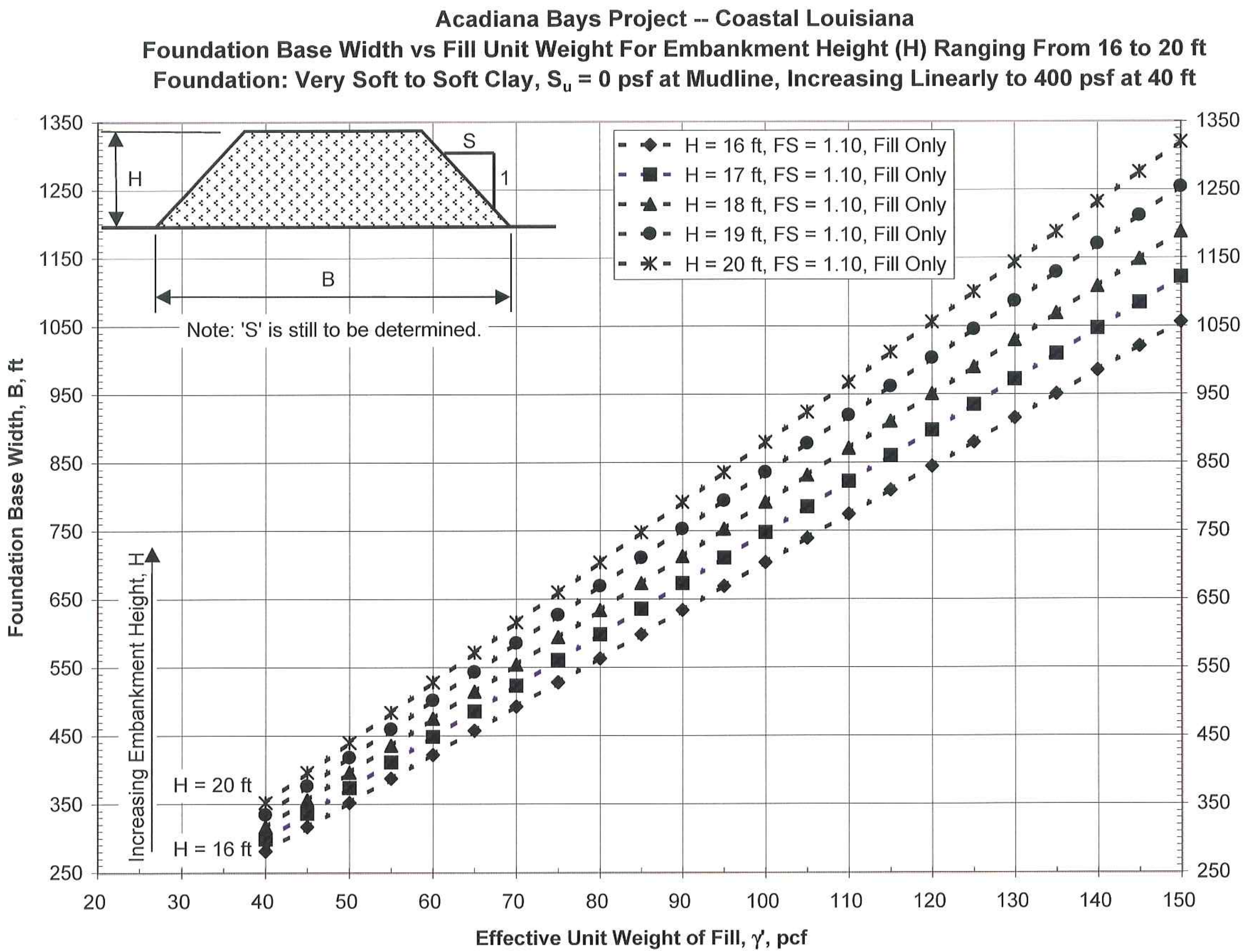
Foundation: Very Soft to Soft Clay, S_u = 0 psf at Mudline, Increasing Linearly to 400 psf at 40 ft




Acadiana Bays Project -- Coastal Louisiana
Foundation Base Width vs Fill Unit Weight For Embankment Height (H) Ranging From 6 to 10 ft
Foundation: Very Soft to Soft Clay, S_u = 0 psf at Mudline, Increasing Linearly to 400 psf at 40 ft
H
S
1
B
Note: 'S' is still to be determined.
H = 6 ft, FS = 1.10, Fill Only
H = 7 ft, FS = 1.10, Fill Only
H = 8 ft, FS = 1.10, Fill Only
H = 9 ft, FS = 1.10, Fill Only
H = 10 ft, FS = 1.10, Fill Only
H = 10 ft
H = 6 ft
Increasing Embankment Height, H
Foundation Base Width, B, ft
Effective Unit Weight of Fill, γ', pcf
750
650
550
450
350
250
150
50
20
30
40
50
60
70
80
90
100
110
120
130
140
150

Acadiana Bays Project -- Coastal Louisiana
Foundation Base Width vs Fill Unit Weight For Embankment Height (H) Ranging From 11 to 15 ft
Foundation: Very Soft to Soft Clay, S_u = 0 psf at Mudline, Increasing Linearly to 400 psf at 40 ft




Acadiana Bays Project -- Coastal Louisiana
Foundation Base Width vs Fill Unit Weight For Embankment Height (H) Ranging From 16 to 20 ft
Foundation: Very Soft to Soft Clay, S_u = 0 psf at Mudline, Increasing Linearly to 400 psf at 40 ft
H = 16 ft, FS = 1.10, Fill Only
H = 17 ft, FS = 1.10, Fill Only
H = 18 ft, FS = 1.10, Fill Only
H = 19 ft, FS = 1.10, Fill Only
H = 20 ft, FS = 1.10, Fill Only
S
1
H
B
Note: 'S' is still to be determined.
Increasing Embankment Height, H
H = 20 ft
H = 16 ft
Foundation Base Width, B, ft
Effective Unit Weight of Fill, γ', pcf
1350
1250
1150
1050
950
850
750
650
550
450
350
250
20
30
40
50
60
70
80
90
100
110
120
130
140
150